REPORT OF FOUNDATION INVESTIGATION

PROPOSED DOCKING FACILITIES
EAST DUBUQUE, ILLINOIS

FOR THE

APPLE RIVER CHEMICAL COMPANY

4771-005-07

DAMES & MOORE
CONSULTANTS IN APPLIED EARTH SCIENCES

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PARTNERS JAMES B THOMPSON • GEORGE D. LEAL

October 26, 1966

Apple River Chemical Company
P.O. Box D
East Dubuque, Illinois 61025

Attention: Mr. Ken Jury

Gentlemen:

Ten copies of our "Report of Foundation Investigation, Proposed Docking Facilities, East Dubuque, Illinois for the Apple River Chemical Company" are herewith submitted.

The scope of the investigation was planned in collaboration with Messrs. Ken Jury and K. J. Siegert of the Apple River Chemical Company and Messrs. F. D. Troxel and R. J. Rasmussen of the Vern E. Alden Company. Certain design data relative to the Proposed Docking Facilities were provided to us by Mr. Rasmussen.

Preliminary data were provided to Mr. G. W. Warren of the Vern E. Alden Company during the course of the investigation.

Yours very truly,

DAMES & MOORE

James B. Thompson

James B. Thompson

JBT:EFG:mf

cc: 5 - Vern E. Alden Company
173 West Madison Street
Chicago, Illinois 60602

Attention: Mr. R. J. Rasmussen

REPORT OF FOUNDATION INVESTIGATION

PROPOSED DOCKING FACILITIES

EAST DUBUQUE, ILLINOIS

FOR THE

APPLE RIVER CHEMICAL COMPANY

SCOPE

This report presents the results of our foundation investigation performed at the site of the Proposed Docking Facilities to be constructed at East Dubuque, Illinois for the Apple River Chemical Company. The locations of the proposed structures are shown on Plate 1, Plot Plan. The purposes of our investigation were as follows:

1 - To explore the subsurface soil and rock conditions to the depths which will be significantly affected by the proposed cells and bridge piers.

2 - To determine those physical properties of the soil and/or rock which will affect the design and construction of the cells and pier foundations.

3 - To provide the data required for the design of the cells and pier foundations, including bearing pressures, settlements, lateral resistance of the soil and/or rock, and the uplift capacities which may be developed.

4 - To provide recommendations relative to any unusual design or construction techniques which may be dictated by the subsurface conditions.

5 - To provide recommendations relative to alternate types of docking facilities, other than the proposed cells, if it is not feasible to construct the cells due to subsurface conditions.

The results of our field explorations and laboratory tests, which form the basis of our recommendations, are presented in the Appendix to this report.

DESIGN CONSIDERATIONS

We understand that the Proposed Docking Facilities will consist of two rock filled cells constructed in the Mississippi River. A pier supported bridge will be constructed from one of the cells to the shore at the location shown on Plate 1. We further understand that the rock filled cells will be designed to impose a unit load on the order of 5,600 pounds per square foot and that the bridge piers will be designed to support total design loads ranging from approximately 46,000 to 232,000 pounds.

The proposed cells and bridge piers will be required to support vertical downward loads, and to resist lateral forces and vertical uplift forces.

A channel will be dredged in the Mississippi River adjacent to the docking facilities. We understand that the bottom of the channel will be established at elevation 581.0.

SITE CONDITIONS

The subsurface conditions at the locations of the proposed cells and bridge piers were investigated by drilling ten test borings at the locations shown on Plate 1. Borings 6, 8 and 9 were drilled from a barge in the Mississippi River. Borings 2 and 7 were not drilled. The remaining borings, Borings 1N, 1S, 3N, 3S, 4N, 4S, and Boring 5 were drilled on the bluff and through the filled slope adjoining the Mississippi River.

Due to the variation in the surface and subsurface conditions, the site is herein divided into two areas to facilitate a description of the surface and subsurface conditions. Area I includes the railroad tracks and extends west to the location of the proposed cells. Area II includes the bluff area east of the railroad tracks. The location of the boundary between the two areas is shown on the attached Plate 2, Subsurface Section AA.

AREA I:

Surface Conditions:

The water level in the Mississippi River was at approximately elevation 592 during the time of our field investigation, August and September 1966. We understand that the normal pool level of the Mississippi River is at approximately elevation 592 in the vicinity of the site. The highest recorded flood elevation was 612.5 in 1965.

The lower portion of the banks of the river between approximately elevation 592 and approximately elevation 617 are composed of granular fill materials. Two sets of railroad tracks are located on the fill at the locations shown on Plates 1 and 2. Surface vegetation consists essentially of grass and weeds.

Subsurface Conditions:

The subsurface conditions in Area I were investigated by drilling four borings (Borings 5, 6, 8 and 9) at the locations shown on Plate 1. The borings revealed that the upper surface of the river bed ranges in elevation from approximately 592 at the location of Boring 5 to approximately 581 at the location of Boring 9. The river bed is composed of loose silty and sandy soils which extend to elevations ranging from approximately 583 at the location of Boring 5 to approximately 558 at the location of Boring 9. The layers of loose silty and sandy soils are underlain by moderately dense to dense sandy soils to the depths penetrated by the borings.

To assist in visualizing the subsurface conditions, a subsurface section has been prepared and is presented on Plate 2. More detailed descriptions of the soils penetrated by the borings are presented on the Log of Borings in the Appendix to this report.

AREA II:

Surface Conditions:

The route of the proposed bridge follows a natural drainage gully which has developed on the face of the bluff adjoining the river. The ground surface elevation ranges from approximately 626 at the base of the bluff near Boring 4N to approximately 688 near the top of the bluff at the location of Boring 1N. The ground surface rises in an irregular manner with slopes which range from near vertical at some locations to approximately one vertical to three horizontal at other locations. Surface vegetation consists essentially of trees and brush.

The subsurface conditions on the bluff were investigated by drilling six borings at the locations shown on Plate 1. The borings revealed that the bedrock, gray dolomite, is relatively close to the ground surface and is visible in numerous exposures on the face of the bluff. The gray dolomite is generally blanketed by a layer of soil ranging from a few inches in thickness to approximately 25 feet in thickness at the location of Boring 1N.

The soil cover consists essentially of moderately dense sandy and/or silty soils. Occasional boulders were encountered in the soil cover in the vicinity of Borings 4N and 4S near the base of the bluff. The soil cover is underlain by gray dolomite to the depths penetrated by the borings. The gray dolomite is weathered and fractured and contains some soil layers in the vicinity of Borings 1N, 1S, 3N and 3S to depths ranging from approximately one foot to nine feet below the upper surface of the gray dolomite. The gray dolomite in the vicinity of Borings 4N and 4S, while weathered to some degree, is essentially sound based on the percent of core recovered.

Based on discussions with personnel of the State Geologic Survey, the gray dolomite is near horizontally bedded, is massive, and is resistive to weathering.

DISCUSSION AND RECOMMENDATIONS

GENERAL:

The Proposed Docking Facilities and bridge piers are treated separately in this section of the report. Several types of foundations are considered suitable for the support of the proposed structures. The choice of the foundation types to be utilized will be dependent on structural as well as economic considerations, which are beyond the scope of this report. Data pertaining to the design and installation of the foundations are presented in a subsequent section, DESIGN DATA, to permit the evaluation of the various foundation types.

Certain factors associated with the design and installation of the foundations are presented in the following section, DESIGN FACTORS. It is considered that a discussion of these factors is necessary to evaluate the data and recommendations presented in the concluding section, DESIGN DATA.

DESIGN FACTORS:

Factors which must be considered in the design and installation of foundations are the variable water level of the Mississippi River, the souring action of the river, the uplift capacity and lateral resistance to movement of foundations, the irregular rock surface, the presence of boulders in the brown sand and gravel at the location of Borings 4N, 4S, 5 and 6, and the natural drainage gully on the face of the bluff.

Variable Water Level Of The Mississippi River:

The water level of the Mississippi River is variable and is dependent on run-off conditions and the operation of the control dams. The low water elevation is 590.3, the high water elevation is 610.9 and the maximum recorded flood elevation was 612.5. It is considered that the variable depth of water may influence the construction period insofar as dredging may be required to utilize barge mounted pile driving equipment in the vicinity of and east of Boring 6.

Scouring Action Of The Mississippi River:

The river bed is composed of loose silty and sandy soils which possess low strength characteristics. It is considered possible that approximately four to six feet of loose silty and sandy soils may be temporarily removed from around the docking facilities and bridge piers during flood periods.

Uplift Capacity And Lateral Resistance To Movement of Foundations:

Due to the scouring action of the Mississippi River, which may temporarily remove approximately four to six feet of the loose silty and sandy soils, it is considered that the foundations must be designed to develop uplift capacity and lateral resistance to movement by greater penetration into the river bed than would be required if there were no scouring action.

Foundations established on the sound limestone at relatively shallow depths on the face of the bluff will require an anchorage system.

Irregular Rock Surface:

Irregularities in the surface of the rock on the face of the slopes may cause certain construction problems. Exposures of the rock surface indicate that the rock is stratified and breaks essentially vertically. It is recommended that foundations established on the rock be located such that the rock at the base of the foundation extends outside all edges of the foundation for a lateral distance of at least six feet.

Boulders In The Brown Sand And Gravel Stratum:

Due to the boulders in the stratum of brown sand and gravel located in the vicinity of and east of Boring 6, it is not considered feasible to utilize wood piles for the support of the proposed bridge piers in this area.

Natural Drainage Gully On The Face Of The Bluff:

To prevent scouring of the soils in the vicinity of foundations established near the gully on the face of the slope, drainage facilities will be required to divert water away from the foundations.

DESIGN DATA:

Docking Facilities:

It is recommended that rock filled steel sheet pile cells be utilized for the Proposed Docking Facilities.

Due to the loose conditions of the layers of silty and sandy soils comprising the upper stratum of the river bed, it is recommended that the steel sheet piles be driven into the underlying denser stratum of gray sand to a tip elevation of 550. Subsequent to driving the sheets and prior to placing the rock fill, the loose soil inside the sheets should be removed to elevation 575 to reduce settlement of the rock fill.

The sheet pile cells will be subjected to interior lateral pressures induced by the rock fill and the bridge load. These pressures will be resisted by interlock tension and by the surrounding soil below the scoured mud line. If desired, we will be pleased to provide you with diagrams showing the distribution of the lateral pressures at such time as the bridge load is provided to us.

Vertical Downward Loads - Sheet pile cells constructed in accordance with the foregoing recommendations may be safely designed to impose a bearing pressure of up to 7,000 pounds per square foot.

The weight of the rock fill plus the bridge load will cause consolidation of the soils inside the sheet pile cells and to lesser degree consolidation of the soils beneath the sheet pile cells. Based on the results of settlement analyses, it is estimated that the rock fill within each sheet pile cell will undergo eventual settlement on the order of three inches due to the weight of the rock fill.

It is estimated that approximately 90 percent of this settlement will occur within three months after placing the rock fill. It is further estimated that the rock fill will undergo additional settlement on the order of one-half inch due to the bridge load. It is estimated that this additional settlement will occur within approximately three months after applying the bridge load.

If the above estimated settlements are unacceptable, they may be reduced to total settlements, (under the weight of the rock fill and bridge loads), on the order of one-quarter inch by removing the loose silty and sandy soils within the sheet pile cells to elevation 558 and replacing these soils with rock fill.

Vertical Uplift Loads - Assuming that the bridge support is structurally tied to the steel sheet pile cell, vertical uplift loads will be resisted by the weight of the bridge and sheet pile cells and frictional resistance exerted both on the interior and exterior sides of the cell. The total frictional resistance outside the sheet pile may be computed by utilizing a unit frictional resistance of 3,000 pounds per lineal foot of cell circumference. The frictional resistance inside the sheet pile cell may be computed by utilizing a unit frictional resistance of 10,000 pounds per lineal foot of cell circumference.

Lateral Loads - Lateral loads acting on the sheet pile cells will be resisted by the passive pressure of the soil below the scoured river bed, elevation 575. The ultimate passive pressure which may be developed can be computed by considering the soils to act as equivalent fluids with the following densities.

ELEVATION INCREMENT FEET-FEET	DENSITY OF EQUIVALENT FLUID POUNDS PER CUBIC FOOT
575 - 560	120
Below 560	180

It is suggested that a suitable factor of safety, possibly 1.5 be applied to the above value for design purposes.

Proposed Bridge Piers:

Two types of foundations are considered suitable for the support of the proposed bridge piers. These types are pile foundations in Area I located west of the existing railroad tracks and conventional spread foundations in Area II located east of the railroad tracks.

Pile Foundations:

Due to the presence of boulders in the sandy soils in the vicinity of Borings 5 and 6, it is recommended that pile foundations be constructed utilizing steel H-piles or concrete filled pipe piles.

Vertical Downward Loads - Pile capacities have been computed for 12 inch steel H-piles and 12 inch diameter concrete filled pipe piles driven "closed-end." Capacities for other sizes can be provided if required.

PILE TYPE	TIP ELEVATION FEET	ULTIMATE SUPPORTING CAPACITY-TONS
12" Steel H	540	25
	520	60
	500	115
12" Concrete Filled Pipe	540	40
	520	100
	500	180

The above capacities are ultimate values, it is recommended that a suitable factor of safety, possibly 1.5, be utilized for design purposes. It is considered that the allowable stresses in the piles selected may impose more severe limitations than the supporting capacity of the soils.

It is estimated that piles driven to the recommended tip elevations and supporting a total design load of up to 100 tons will undergo settlement on the order of one-half inch or less.

Vertical Uplift Loads - It is recommended that the vertical uplift capacity be considered equal to 50 percent of the vertical downward capacity.

Lateral Loads - Lateral loads acting on the piles will be resisted by the passive pressure of the soil below the scoured river bed, elevation 575. The ultimate passive pressure which may be developed by the piles can be computed by considering the soils to act as equivalent fluids with the following densities.

PILE TYPE	ELEVATION INCREMENT FEET-FEET	DENSITY OF EQUIVALENT FLUID POUNDS PER CUBIC FOOT
12" Steel H	575 - 560	120
	Below 560	180
12" Concrete Filled Pipe	575 - 560	120
	Below 560	200

Spread Foundations:

Due to the thickness, (up to 25 feet), of soil overlying the gray dolomite in the vicinity of Borings 1N and 1S, it is recommended that the bridge piers in this area be supported on spread foundations established in the natural sandy soils at a minimum depth of at least six feet below the final planned grade. It is considered that a foundation located in the vicinity of Boring 1N will be established at approximately elevation 680, or approximately eight feet below the existing cut grade to attain the elevation of the sand deposit.

It is recommended that the bridge piers in the vicinity of Borings 3N, 3S, 4N and 4S be established on the gray dolomite regardless of the degree of weathering. However, it is recommended that foundations established in the gray dolomite be anchored to the dolomite with grouted reinforcing bars extending at least ten feet below the bottom of the foundation and extending at least five feet below the bottom of the weathered dolomite, whichever results in the longer bars. It is further recommended that spread foundations established on the gray dolomite be located at a depth such that the bearing surface of the rock extends a lateral distance of at least six feet outside the edge of the foundation in all directions.

Vertical Downward Loads - Spread foundations established in the sand may be proportioned to impose a bearing pressure of up to 3,000 pounds per square foot. This pressure refers to the total design loads, dead and live, and is a net pressure. Therefore, the weight of the foundations and backfill may be ignored in proportioning the foundations. It is estimated that spread foundations established in the sand stratum at a minimum depth of six feet below the final planned grade and supporting a total design load of up to 50,000 pounds will undergo settlement on the order of one-quarter inch or less.

Spread foundations established on the gray dolomite may be designed to impose a net bearing pressure of up to 10,000 pounds per square foot. To prevent excessive effects of frost action, it is recommended that four feet of soil cover be provided over each foundation. It is estimated that anchored spread foundations established on the gray dolomite and supporting a total design load of up to approximately 300,000 pounds will undergo settlement of less than one-quarter inch.

Vertical Uplift Loads - Vertical uplift loads will be resisted by the weight of the foundations, backfill, and the bridge. It is recommended that the backfill be composed of thoroughly compacted granular material. The unit weight of the backfill may be assumed to be 120 pounds per cubic foot. Additional resistance to uplift loads will be provided by the rock anchors. This additional resistance will depend on the number, length and diameter of the grouted anchors.

Lateral Loads - It is recommended that spread foundations established in the sand stratum be backfilled with thoroughly compacted granular material. Lateral loads will be resisted by passive pressures developed on the side of the foundation opposite the load and friction between the foundation and sand. The allowable passive pressure reacting on one side of the foundation may be computed by considering the natural sand and compacted granular backfill to act as an equivalent fluid with a density of 250 pounds per cubic foot. The coefficient of friction between the sand and concrete may be considered equal to 0.5.

Spread foundations established in the gray dolomite will resist lateral loads by developing passive resistance in the rock or passive resistance in a compacted granular backfill if the foundations are established on top of the rock, by developing friction between the foundation and the rock, and by the rock anchors. The allowable passive resistance which may be developed in the gray dolomite may be considered equal to 500 pounds per square foot. The allowable passive resistance which may be developed in a compacted granular backfill may be computed as described in the preceding paragraph. The coefficient of friction between the foundation and gray dolomite may be considered equal to 0.6. The lateral resistance provided by the rock anchors will depend on the diameter and number of anchors.

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The following Plates and Appendix are attached and complete this report:

Plate 1 - Plot Plan

Plate 2 - Subsurface Section A-A

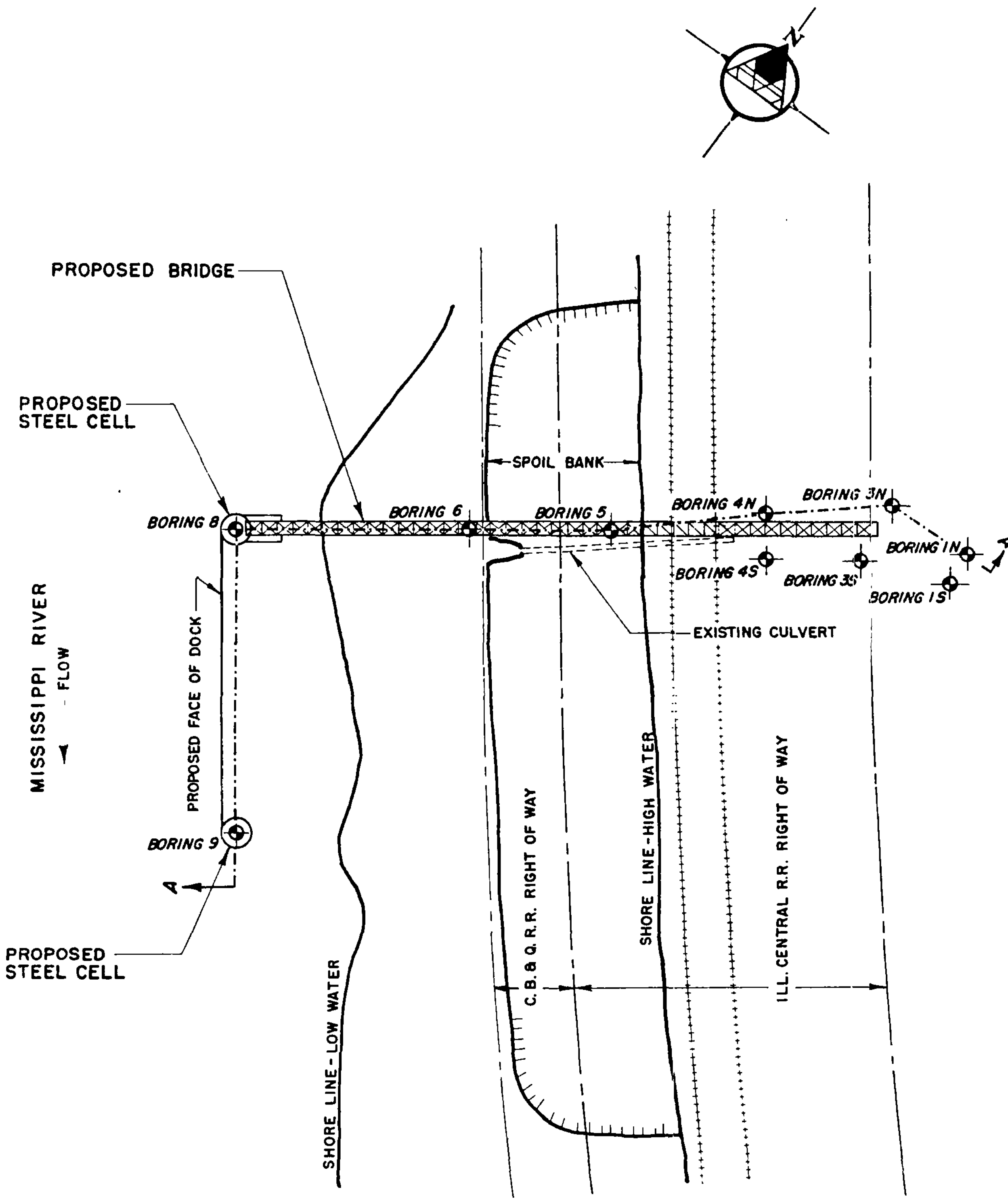
Appendix - Explorations And Laboratory Tests

Respectfully submitted,

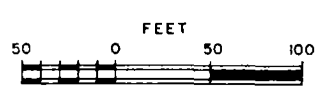
DAMES & MOORE

James B. Thompson

James B. Thompson



PLOT PLAN



DRAWING REFERENCE:
PROPOSED DOCK ON THE MISSISSIPPI RIVER
NEAR EAST DUBUQUE, COUNTY OF JoDAVIESS, STATE-ILLINOIS
APPLICATION BY APPLE RIVER CHEMICAL CO.
SHEET 2 OF 3 - DATED 5/18/66
PREPARED BY VERN E. ALDEN CO.
SOIL BORING LOCATIONS

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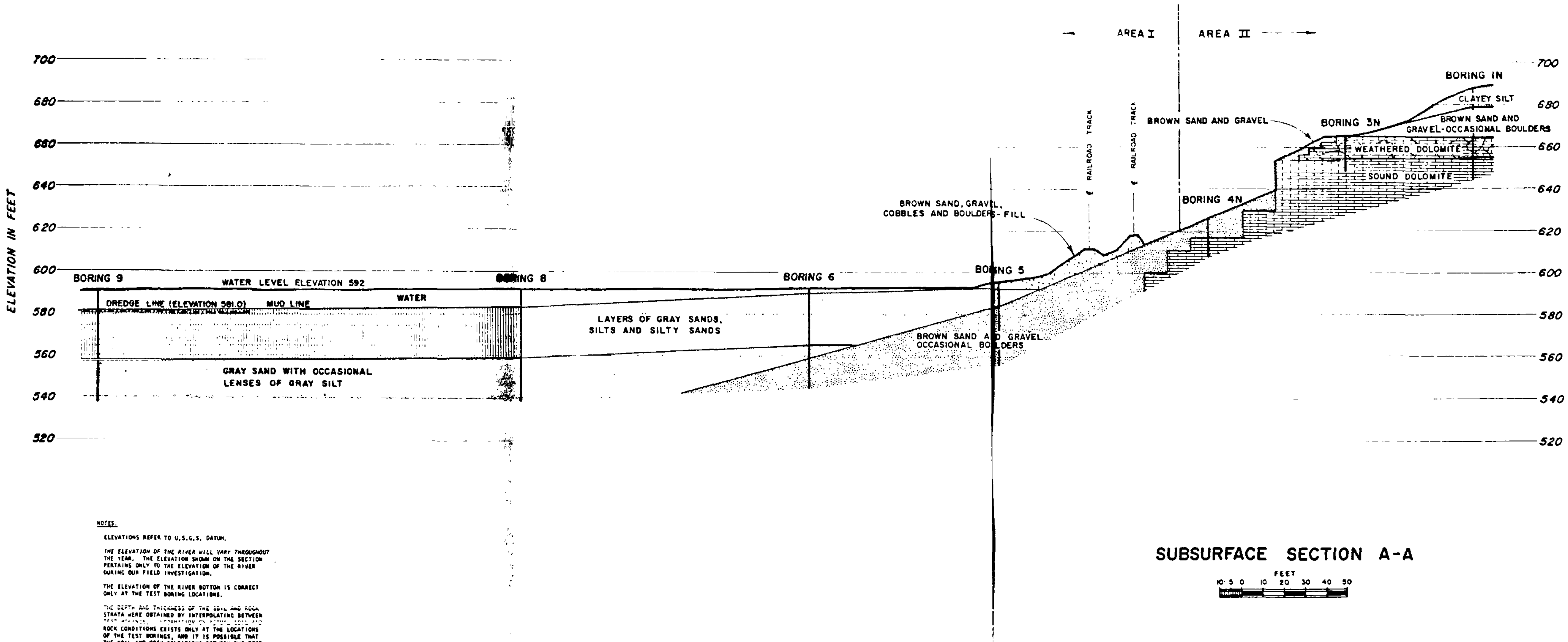
PLATE I



SUBSURFACE SECTION A-A

NOTES.

ELEVATIONS REFER TO U.S.G.S. DATUM.

THE ELEVATION OF THE RIVER WILL VARY THROUGHOUT THE YEAR. THE ELEVATION SHOWN ON THE SECTION PERTAINS ONLY TO THE ELEVATION OF THE RIVER DURING OUR FIELD INVESTIGATION.

THE ELEVATION OF THE RIVER BOTTOM IS CORRECT ONLY AT THE TEST BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SOIL AND ROCK STRATA WERE OBTAINED BY INTERPOLATING BETWEEN TEST BORINGS. INFORMATION ON ACTUAL SOIL AND ROCK CONDITIONS EXISTS ONLY AT THE LOCATIONS OF THE TEST BORINGS, AND IT IS POSSIBLE THAT THE SOIL AND ROCK CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

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PLATE 2

Appendix

APPENDIX

EXPLORATIONS AND LABORATORY TESTS

EXPLORATIONS:

The subsurface conditions at the site were investigated by drilling ten test borings with barge mounted rotary wash type drilling equipment, truck mounted rotary wash type drilling equipment, and portable hand operated drilling equipment to depths ranging from approximately 11 feet to 53 feet. Graphical representations of the soils and rock penetrated by the borings are shown on Plates A-1A through A-1G, Log of Borings. The method utilized in classifying the soils is defined by the Unified Soil Classification System presented on Plate A-2. Undisturbed samples of the various soil strata penetrated by the borings were obtained in a soil sampler of the type illustrated on Plate A-3, Soil Sampler Type U. Rock was cored in certain of the borings as indicated on the Log of Borings. The percent of core recovered is presented to the left of the Log of Borings. The borings were located in the field by a survey crew from the firm of Bartels, McMahan and LeMay Engineering Company from Dubuque, Iowa.

LABORATORY TESTS:

Laboratory tests were performed to determine the strength and compressibility characteristics of certain soil strata. In addition, moisture and density tests were performed for correlation purposes.

Strength Tests:

Direct shear tests were performed on selected soil samples to determine the strength characteristics of the various soil strata penetrated by the borings. The direct shear tests were performed in the manner described or Plate A-4, Method of Performing Direct Shear And Friction Tests. A stress-strain curve was plotted for each test and the shearing strength of each soil type tested was determined from these curves. The results of the strength tests are presented to the left of the Log of Borings in the manner described by the Key to Test Data shown on Plate A-2.

Consolidation Test:

One consolidation test was performed on a selected sample of the silty and sandy soils to provide data for estimating settlement of the cells. The consolidation test was performed in the manner described on Plate A-5, Method of Performing Consolidation Tests. The results of the consolidation test are presented on Plate A-6, Consolidation Test Data.

Moisture And Density Tests:

Moisture and density tests were performed in conjunction with each strength and consolidation test. Additional moisture and density tests were performed for correlation purposes. The results of the moisture and density tests are presented to the left of the Log of Borings in the manner described by the Key to Test Data shown on Plate A-2.

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The following Plates are attached and complete this Appendix:

Plate A-1A - Log of Borings (Boring 1N)

Plate A-1B - Log of Borings (Borings 1S and 3N)

Plate A-1C - Log of Borings (Borings 3S, 4N and 4S)

Plate A-1D - Log of Borings (Boring 5)

Plate A-1E - Log of Borings (Boring 6)

Plate A-1F - Log of Borings (Boring 8)

Plate A-1G - Log of Borings (Boring 9)

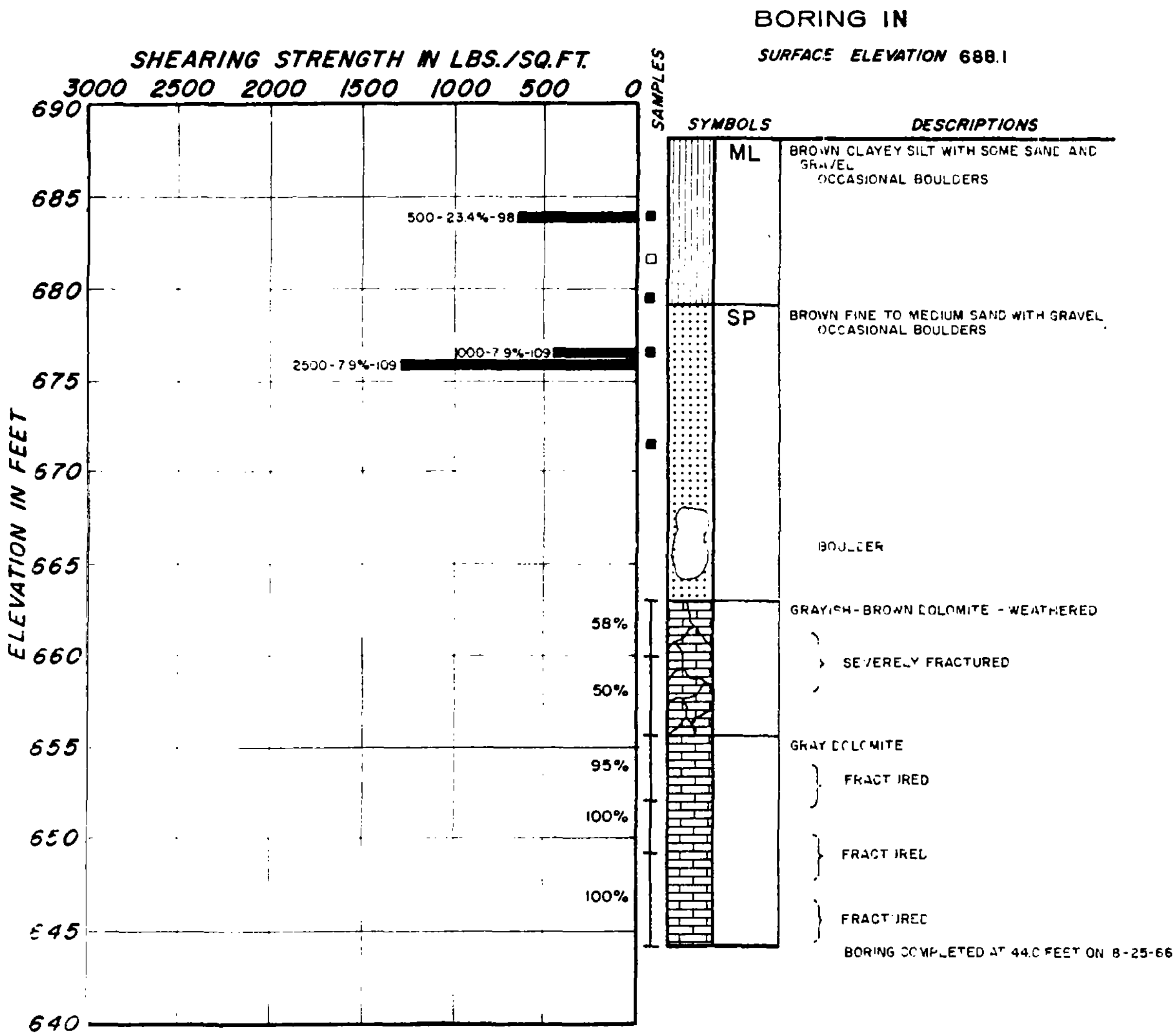
Plate A-2 - Unified Soil Classification System and Key to Test Data

Plate A-3 - Soil Sampler Type U

Plate A-4 - Method of Performing Direct Shear and Friction Tests

Plate A-5 - Method of Performing Consolidation Tests

Plate A-6 - Consolidation Test Data



LOG OF BORINGS

NOTE:

ELEVATIONS REFER TO U.S.G.S. DATUM.

100% INDICATES DEPTH, LENGTH AND PERCENT OF CORE RUN RECOVERED.

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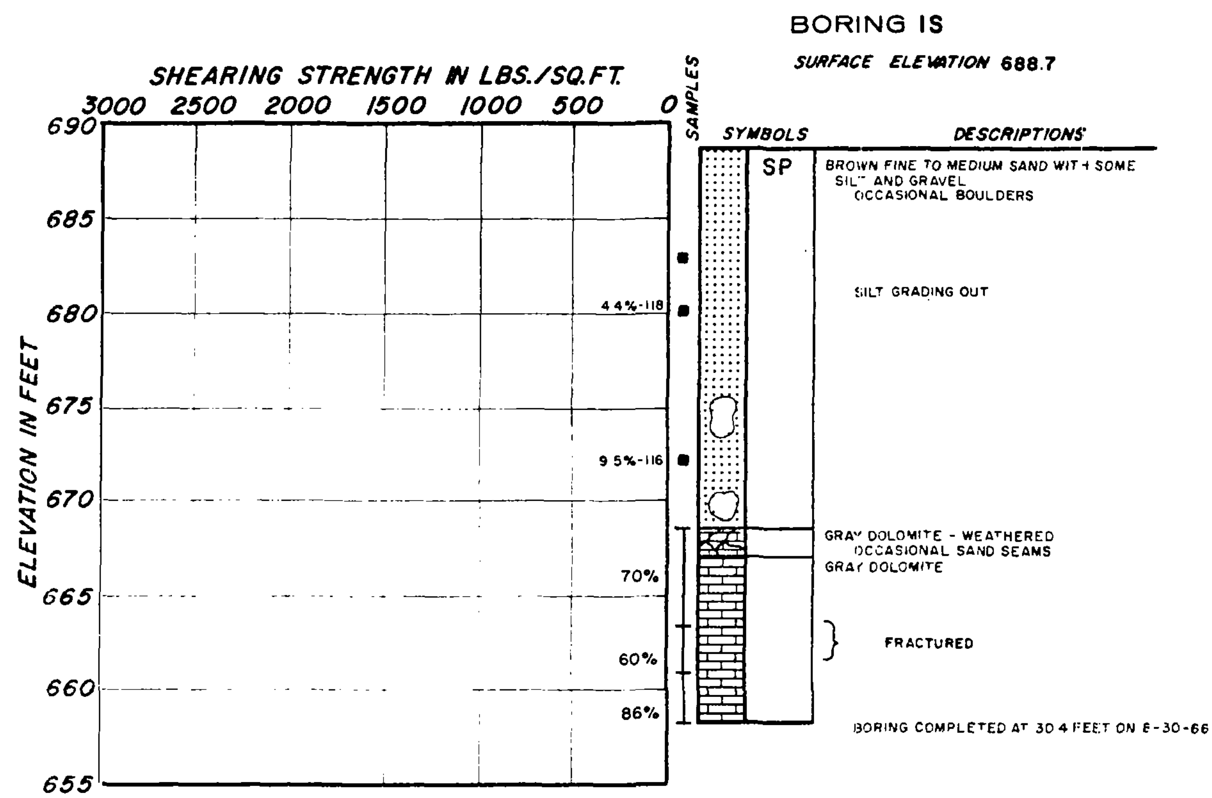

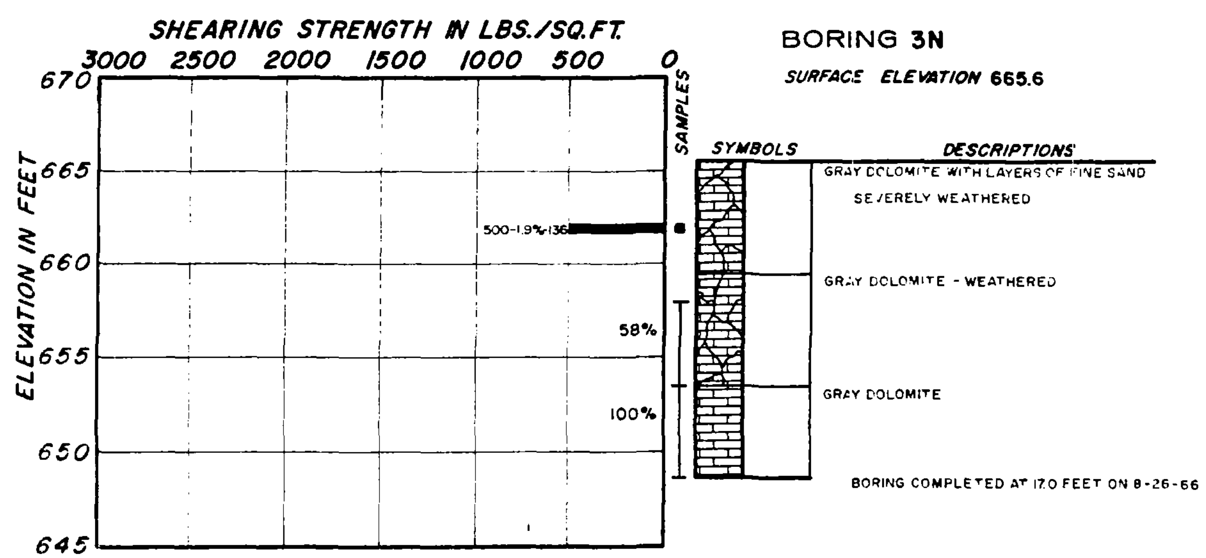

LOG OF BORINGS

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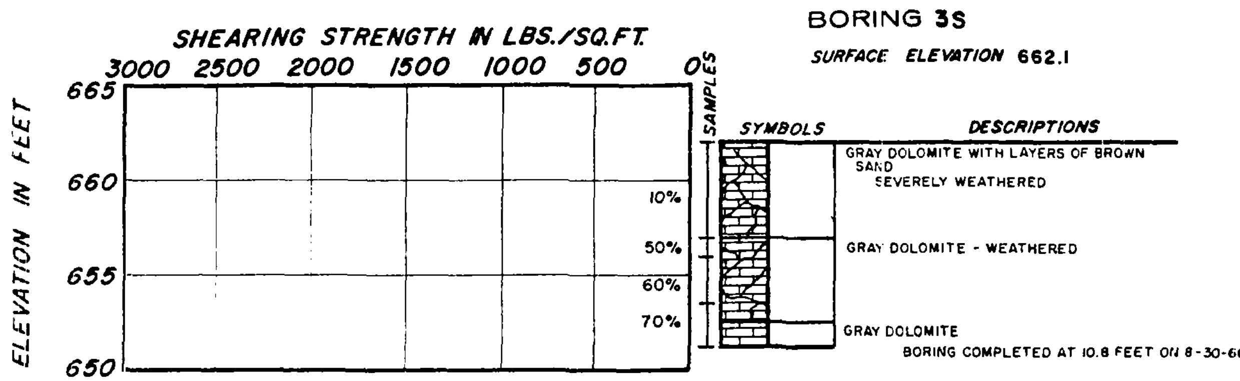

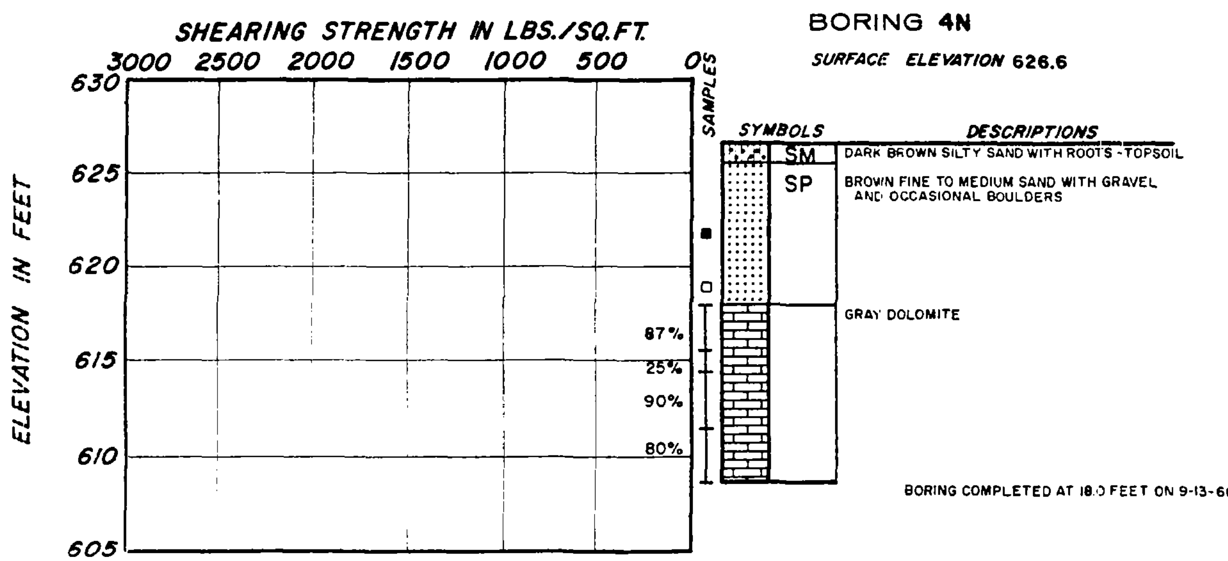

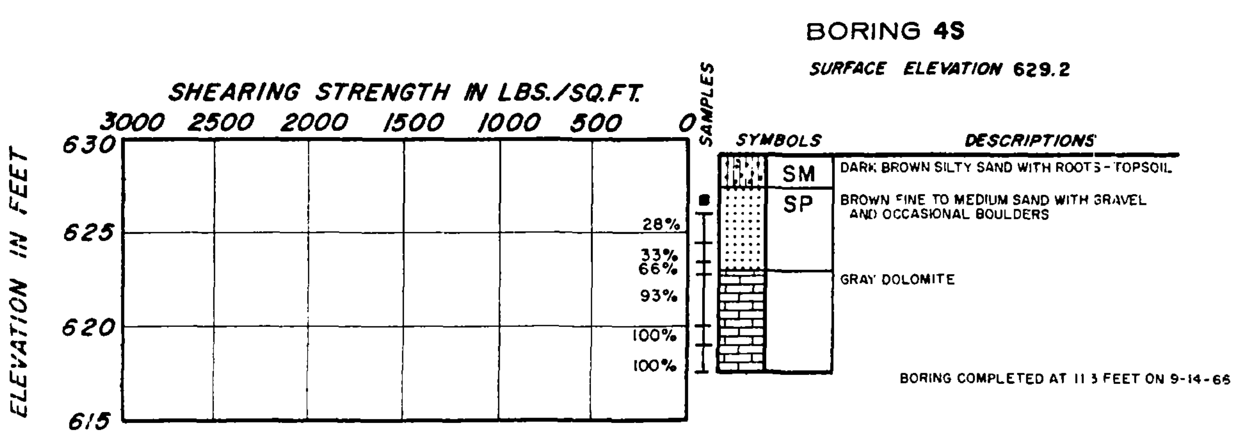

LOG OF BORINGS

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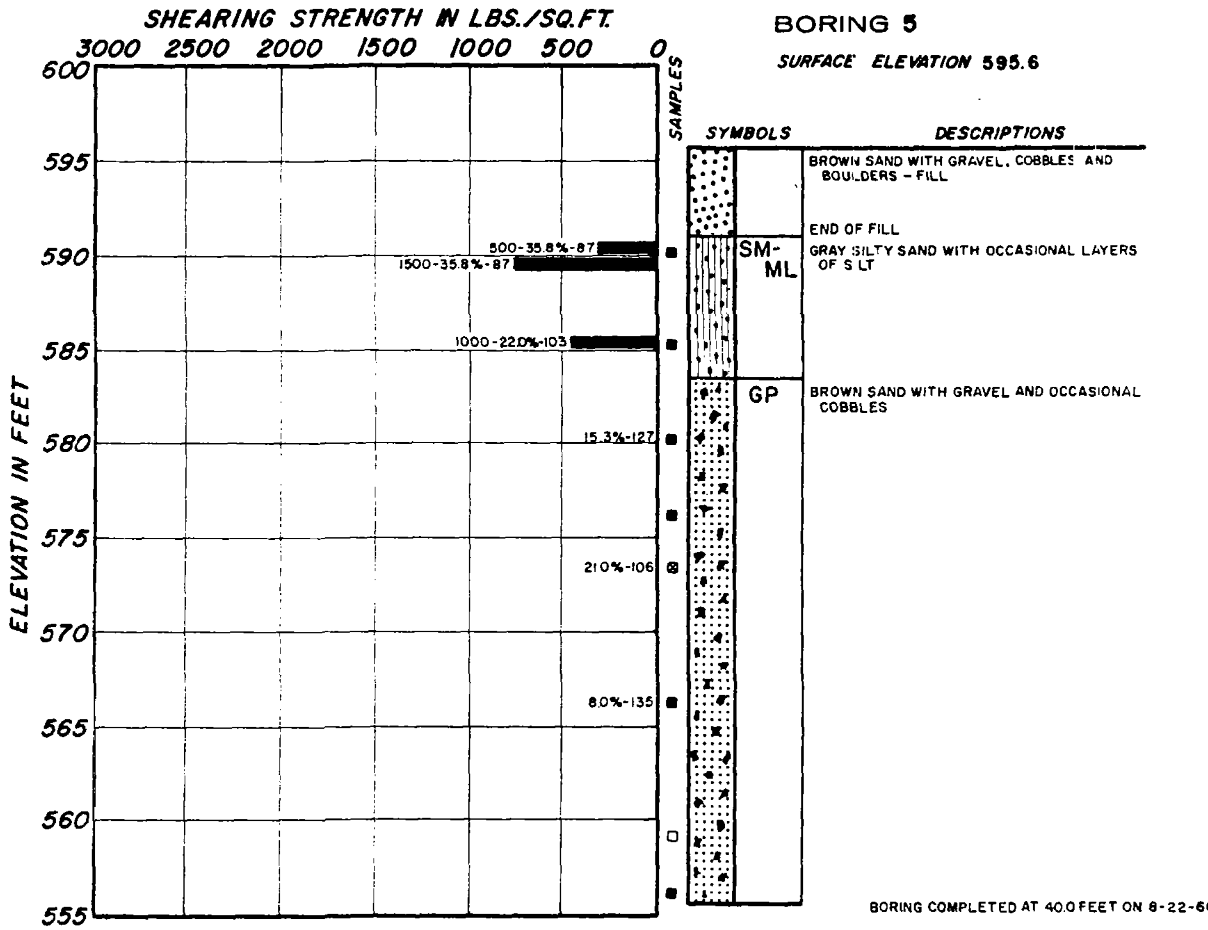
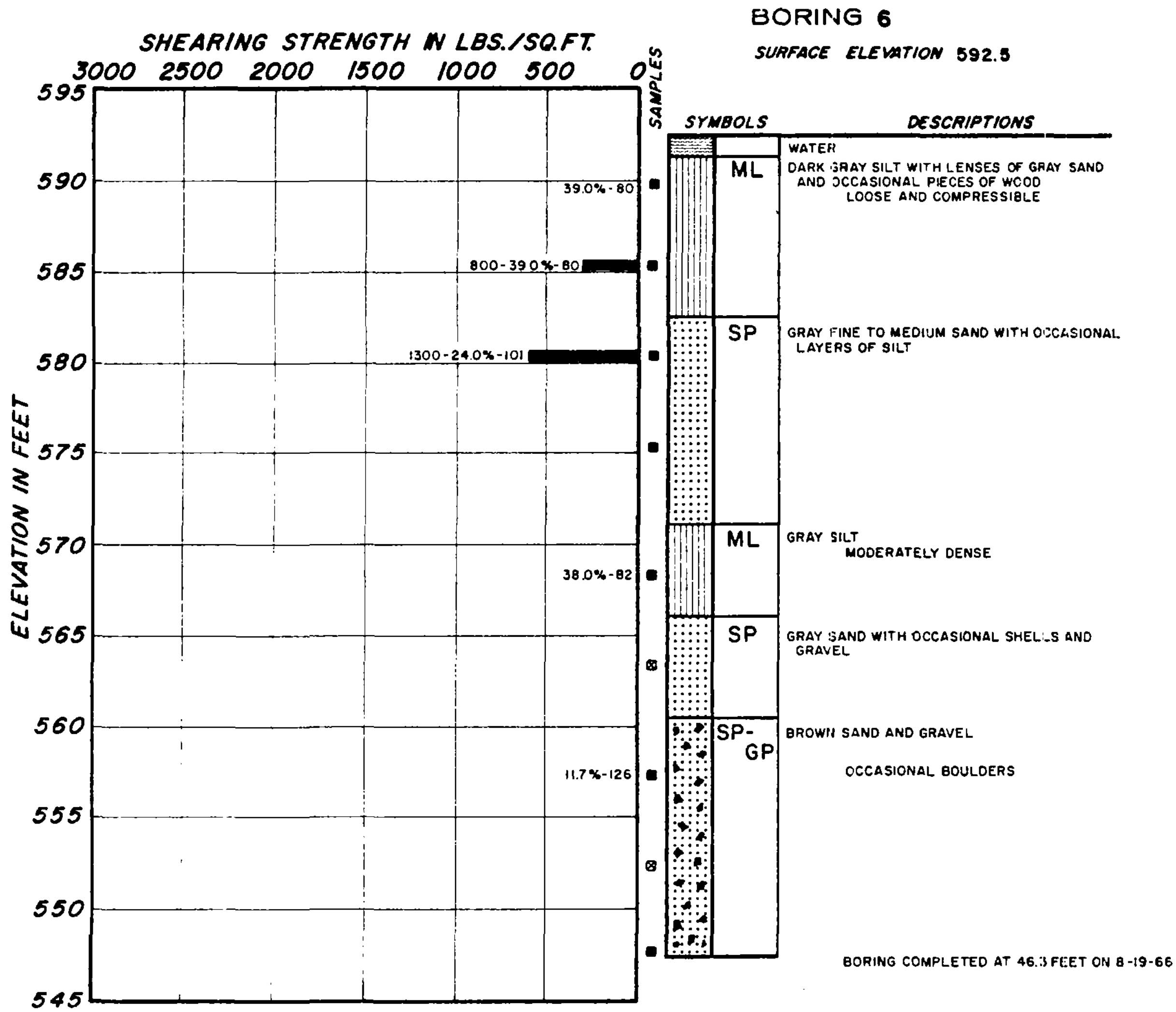


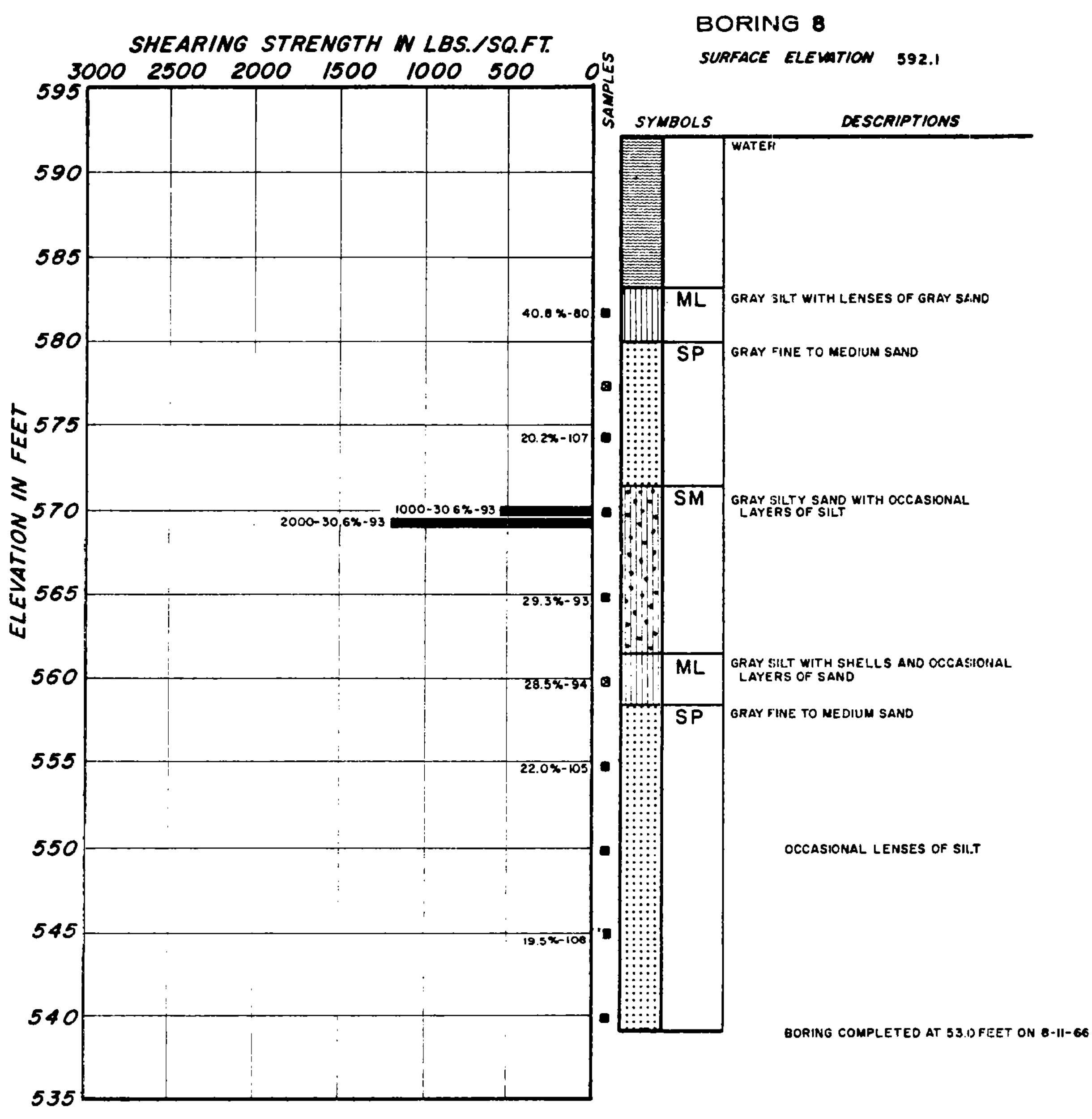
LOG OF BORINGS

DAMES & MOORE



LOG OF BORINGS

DAMES & MOORE



LOG OF BORINGS

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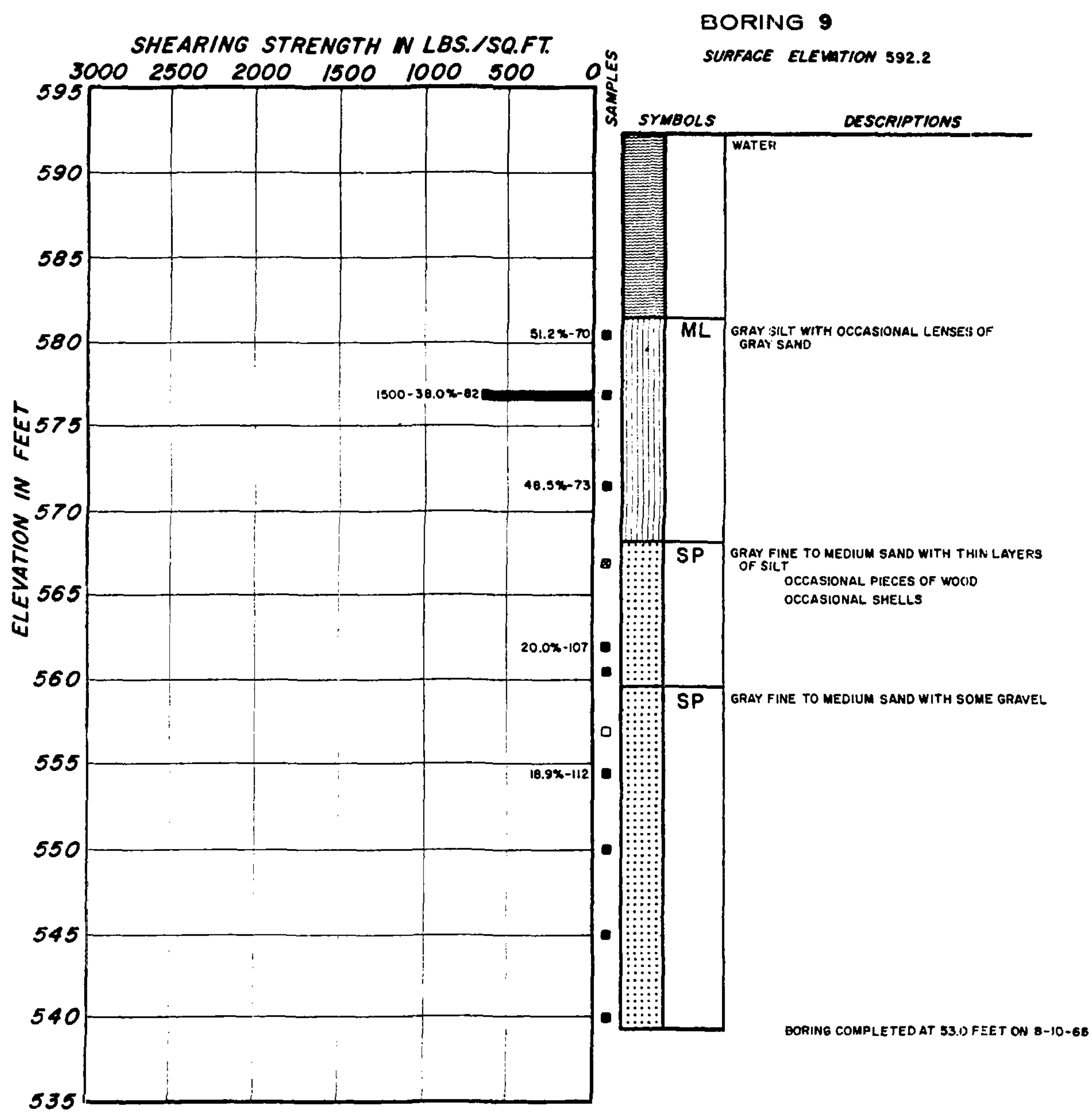
PLATE A-1F



LOG OF BORINGS

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MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVEL (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
				MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

SOIL CLASSIFICATION CHART

SHEARING STRENGTH IN POUNDS PER SQUARE FOOT

FRICTION OF SOIL ON STEEL IN POUNDS PER SQUARE FOOT

UNCONFINED COMPRESSION TESTS

DRY DENSITY EXPRESSED IN POUNDS PER CUBIC FOOT

SHEARING STRENGTH IN POUNDS PER SQUARE FOOT

TRIAXIAL COMPRESSION TESTS

TEST NORMAL PRESSURE IN POUNDS PER SQUARE FOOT (FROM MOHR CIRCLE)

DRY DENSITY EXPRESSED IN POUNDS PER CUBIC FOOT

SHEARING STRENGTH IN POUNDS PER SQUARE FOOT (FROM MOHR CIRCLE)

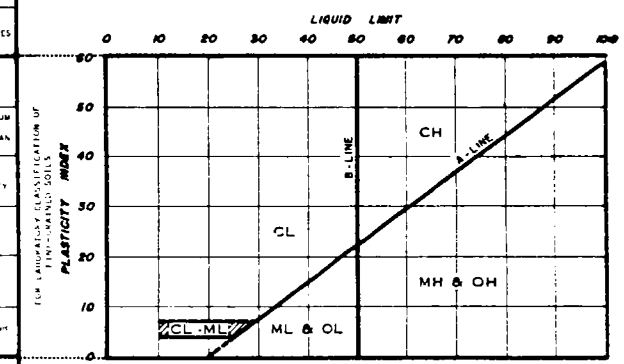
ROCK COMPRESSION TESTS

COMPRESSIVE STRENGTH IN POUNDS PER SQUARE INCH

KEY TO TEST DATA

- INDICATES DEPTH OF UNDISTURBED SAMPLE
- INDICATES DEPTH OF DISTURBED SAMPLE
- INDICATES DEPTH OF SAMPLING ATTEMPT WITH NO RECOVERY
- INDICATES DEPTH OF SPLIT-SPOON SAMPLE
- INDICATES DEPTH AND LENGTH OF CORING RUN

KEY TO SAMPLES



PLASTICITY CHART

UNIFIED SOIL CLASSIFICATION SYSTEM

DAMES & MOORE

PLATE A-2

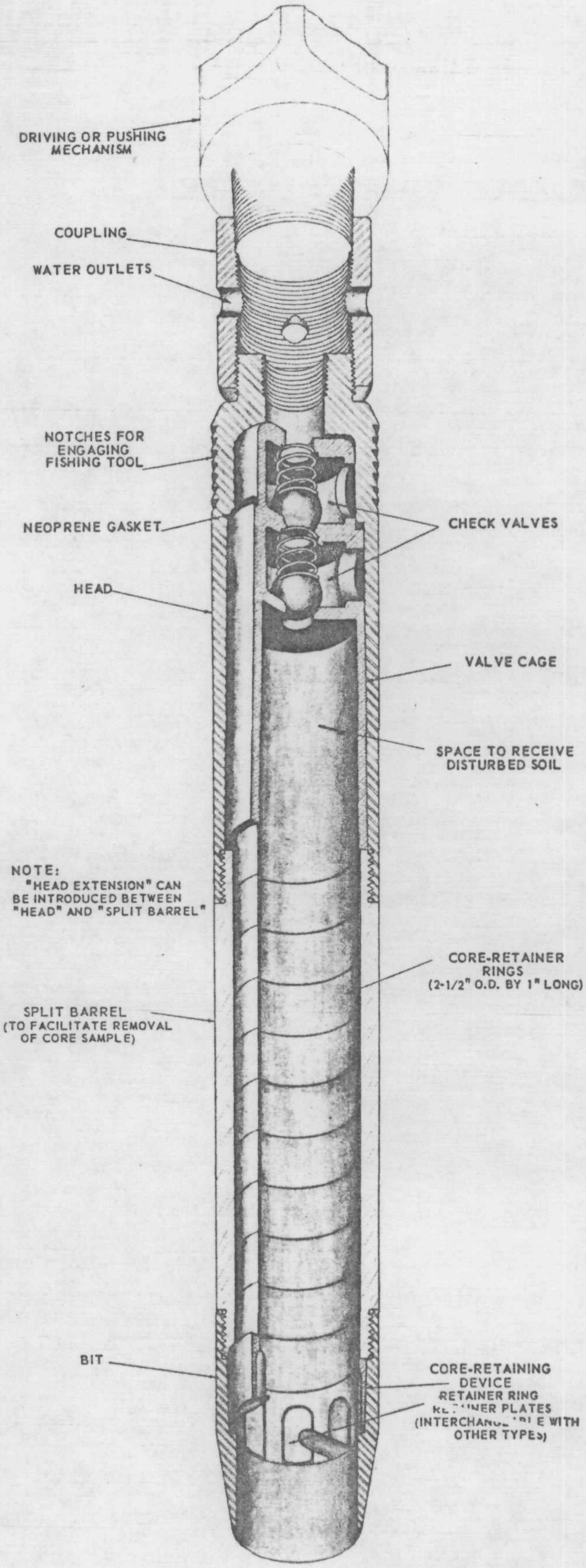
DAMES & MOORE
APPLIED EARTH SCIENCES

PLATE A-3

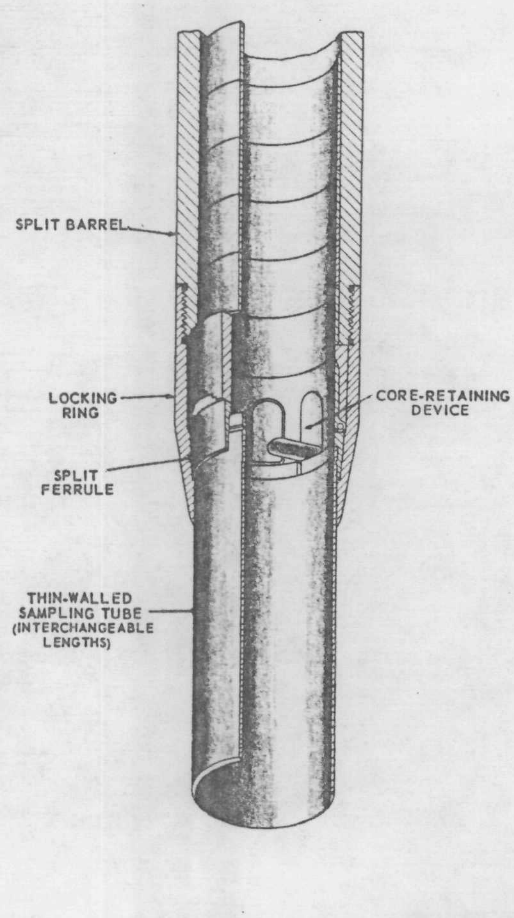
SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER

U. S. PATENT NO. 2,318,062

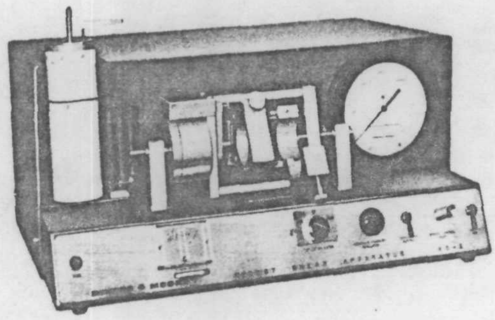


ALTERNATE ATTACHMENTS



Method Of Performing Direct Shear And Friction Tests

DIRECT SHEAR TESTS ARE PERFORMED TO DETERMINE THE SHEARING STRENGTHS OF SOILS. FRICTION TESTS ARE PERFORMED TO DETERMINE THE FRICTIONAL RESISTANCES BETWEEN SOILS AND VARIOUS OTHER MATERIALS SUCH AS WOOD, STEEL, OR CONCRETE. THE TESTS ARE PERFORMED IN THE LABORATORY TO SIMULATE ANTICIPATED FIELD CONDITIONS.



DIRECT SHEAR TESTING
& RECORDING APPARATUS

EACH SAMPLE IS TESTED WITHIN THREE BRASS RINGS, TWO AND ONE-HALF INCHES IN DIAMETER AND ONE INCH IN LENGTH. UNDISTURBED SAMPLES OF IN-PLACE SOILS ARE TESTED IN RINGS TAKEN FROM THE SAMPLING DEVICE IN WHICH THE SAMPLES WERE OBTAINED. LOOSE SAMPLES OF SOILS TO BE USED IN CONSTRUCTING EARTH FILLS ARE COMPACTED IN RINGS TO PREDETERMINED CONDITIONS AND TESTED.

Direct Shear Tests

A THREE-INCH LENGTH OF THE SAMPLE IS TESTED IN DIRECT DOUBLE SHEAR. A CONSTANT PRESSURE, APPROPRIATE TO THE CONDITIONS OF THE PROBLEM FOR WHICH THE TEST IS BEING PERFORMED, IS APPLIED NORMAL TO THE ENDS OF THE SAMPLE THROUGH POROUS STONES. A SHEARING FAILURE OF THE SAMPLE IS CAUSED BY MOVING THE CENTER RING IN A DIRECTION PERPENDICULAR TO THE AXIS OF THE SAMPLE. TRANSVERSE MOVEMENT OF THE OUTER RINGS IS PREVENTED.

THE SHEARING FAILURE MAY BE ACCOMPLISHED BY APPLYING TO THE CENTER RING EITHER A CONSTANT RATE OF LOAD, A CONSTANT RATE OF DEFLECTION, OR INCREMENTS OF LOAD OR DEFLECTION. IN EACH CASE, THE SHEARING LOAD AND THE DEFLECTIONS IN BOTH THE AXIAL AND TRANSVERSE DIRECTIONS ARE RECORDED AND PLOTTED. THE SHEARING STRENGTH OF THE SOIL IS DETERMINED FROM THE RESULTING LOAD-DEFLECTION CURVES.

Friction Tests

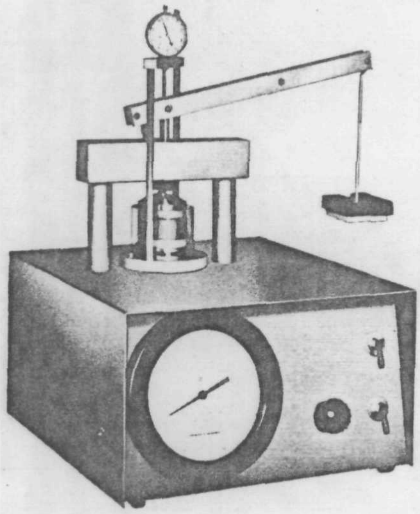
IN ORDER TO DETERMINE THE FRICTIONAL RESISTANCE BETWEEN SOIL AND THE SURFACES OF VARIOUS MATERIALS, THE CENTER RING OF SOIL IN THE DIRECT SHEAR TEST IS REPLACED BY A DISK OF THE MATERIAL TO BE TESTED. THE TEST IS THEN PERFORMED IN THE SAME MANNER AS THE DIRECT SHEAR TEST BY FORCING THE DISK OF MATERIAL FROM THE SOIL SURFACES.

CHECKED BY ______ FILE ______ BY ______ DATE ______

METHOD OF PERFORMING CONSOLIDATION TESTS

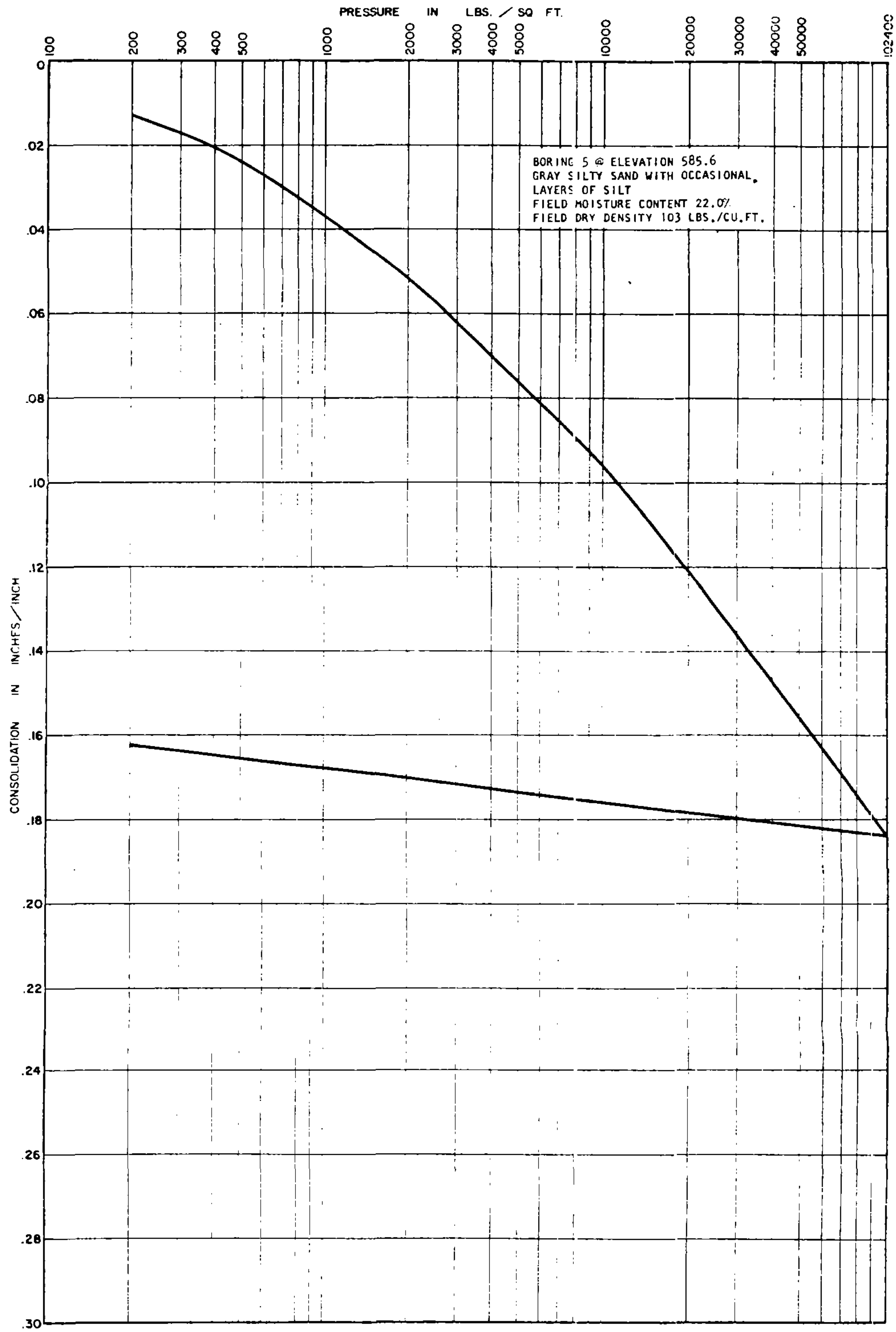
CONSOLIDATION TESTS ARE PERFORMED TO EVALUATE THE VOLUME CHANGES OF SOILS SUBJECTED TO INCREASED LOADS. TIME-CONSOLIDATION AND PRESSURE-CONSOLIDATION CURVES MAY BE PLOTTED FROM THE DATA OBTAINED IN THE TESTS. ENGINEERING ANALYSES BASED ON THESE CURVES PERMIT ESTIMATES TO BE MADE OF THE PROBABLE MAGNITUDE AND RATE OF SETTLEMENT OF THE TESTED SOILS UNDER APPLIED LOADS.

EACH SAMPLE IS TESTED WITHIN BRASS RINGS TWO AND ONE-HALF INCHES IN DIAMETER AND ONE INCH IN LENGTH. UNDISTURBED SAMPLES OF IN-PLACE SOILS ARE TESTED IN RINGS TAKEN FROM THE SAMPLING DEVICE IN WHICH THE SAMPLES WERE OBTAINED. LOOSE SAMPLES OF SOILS TO BE USED IN CONSTRUCTING EARTH FILLS ARE COMPACTED IN RINGS TO PREDETERMINED CONDITIONS AND TESTED.



DEAD LOAD-PNEUMATIC CONSOLIDOMETER

IN TESTING, THE SAMPLE IS RIGIDLY CONFINED LATERALLY BY THE BRASS RING. AXIAL LOADS ARE TRANSMITTED TO THE ENDS OF THE SAMPLE BY POROUS DISKS. THE DISKS ALLOW DRAINAGE OF THE LOADED SAMPLE. THE AXIAL COMPRESSION OR EXPANSION OF THE SAMPLE IS MEASURED BY A MICROMETER DIAL INDICATOR AT APPROPRIATE TIME INTERVALS AFTER EACH LOAD INCREMENT IS APPLIED. EACH LOAD IS ORDINARILY TWICE THE PRECEDING LOAD. THE INCREMENTS ARE SELECTED TO OBTAIN CONSOLIDATION DATA REPRESENTING THE FIELD LOADING CONDITIONS FOR WHICH THE TEST IS BEING PERFORMED. EACH LOAD INCREMENT IS ALLOWED TO ACT OVER AN INTERVAL OF TIME DEPENDENT ON THE TYPE AND EXTENT OF THE SOIL IN THE FIELD.



CONSOLIDATION TEST DATA

DAMES & MOORE